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Analysis

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1 Introduction

The controlling sections of chords in a truss are often influenced by a moment peak, see figure 1. At triangular trusses the maximum peak moment and maximum axial compression force in the top chord often appear at the heel joint.

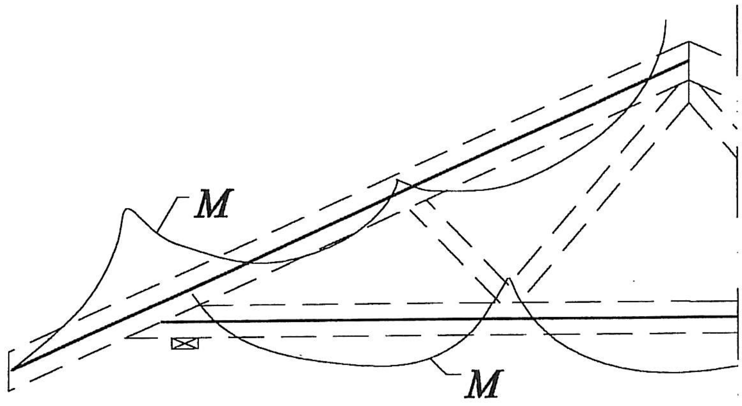


Figure 1: Variation of the moment, M, in the chords of a W-truss.

In order to obtain optimal truss designs different statistical analyses of the wood strength parameters have been made to obtain increased capacity of the sections with moment peak. Another method of increasing the moment capacity is to reinforce the dimension controlling areas with a punched metal plate of same type as used in the joints of the truss, see figure 2. An advantage of this method is that the price of the reinforcement material and the production of the reinforcement are very limited compared to the total costs of a truss. By introducing the reinforcement it may be possible to reduce the cross-sectional depths of the chords.

The reinforcing effect of punched metal plate fasteners in areas with a moment peak is analysed by tests. According to DS/EN 408 the bending capacity (bending strength) of timber shall be determined for beams loaded in 4-point bending. However, the

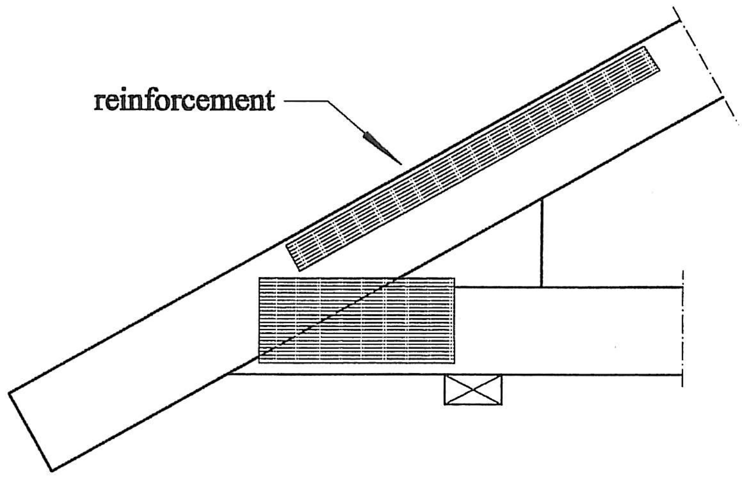


Figure 2: Heel joint with a wedge and a reinforcement.

present work is limited to analysis of the reinforcing effect in areas with moment peaks only. Therefore, the test specimens are made with beams loaded in 3-point bending, as shown in figure 3.

The tests and the test results are described in this paper. The failure types are given and some load-displacement curves are shown. Further, the tests are compared to a numerical model.

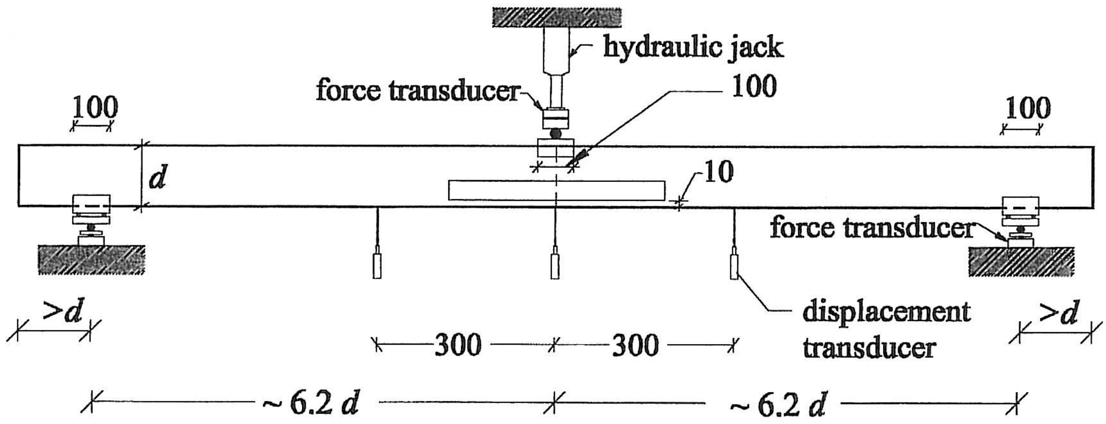


Figure 3: Test arrangement for beams in 3-point bending. Dimensions in mm.

2 Test series

The test programme is shown in table 1. The applied timber is Swedish spruce with 45 mm thickness. The timber was selected and graded[1] by a truss plant.

Series no.	No. of tests	Timber grade[1]	Timber d mm	Timber span mm	Plate width w mm	Plate length l mm
RS1	10	K18	120	1500	-	-
RS3	20	"	170	2100	-	-
RS5	10	K24	170	"	-	-
RS6	10	K18	120	1500	34	776
RS7	10	"	120	"	76	516
RS8	10	"	120	"	34	516
RS9	10	"	170	2100	76	516
RS10	10	"	170	"	34	776
RS12	10	"	170	"	34	516
RS14	10	K24	170	"	34	776
RS15	10	"	170	"	34	516

Table 1: Test program with 11 different series.

The reinforcement is made by GNA 20 S plates from Gang-Nail Systems. The thickness of the plate is 1 mm and the tooth length is about 8 mm. Different widths and lengths of the plates are used. All plates are located at the centre of the beams 10 mm from the tensile edge, see figure 3. The test specimens have been conditioned and manufactured according to prEN 1075. During the test period the moisture content of the timber was about 10-12%.

To obtain a section with a moment peak all samples are loaded in 3-point bending, see figure 3. The distance between the mid load and the reactions is $\sim 6.2d$, where d is the cross-sectional depth of the beam.

According to DS/EN 408 the load is applied at constant loading head movement (deformation controlled) so the maximum load is reached within 300s ($\pm$120s). The loading heads have a width of 100 mm.

In figure 4 the scaled test specimens with a reinforcement are shown.

[1]Strength classes K18 and K24 can be transmitted to S8 and S10, according to ECE Timber Committee.

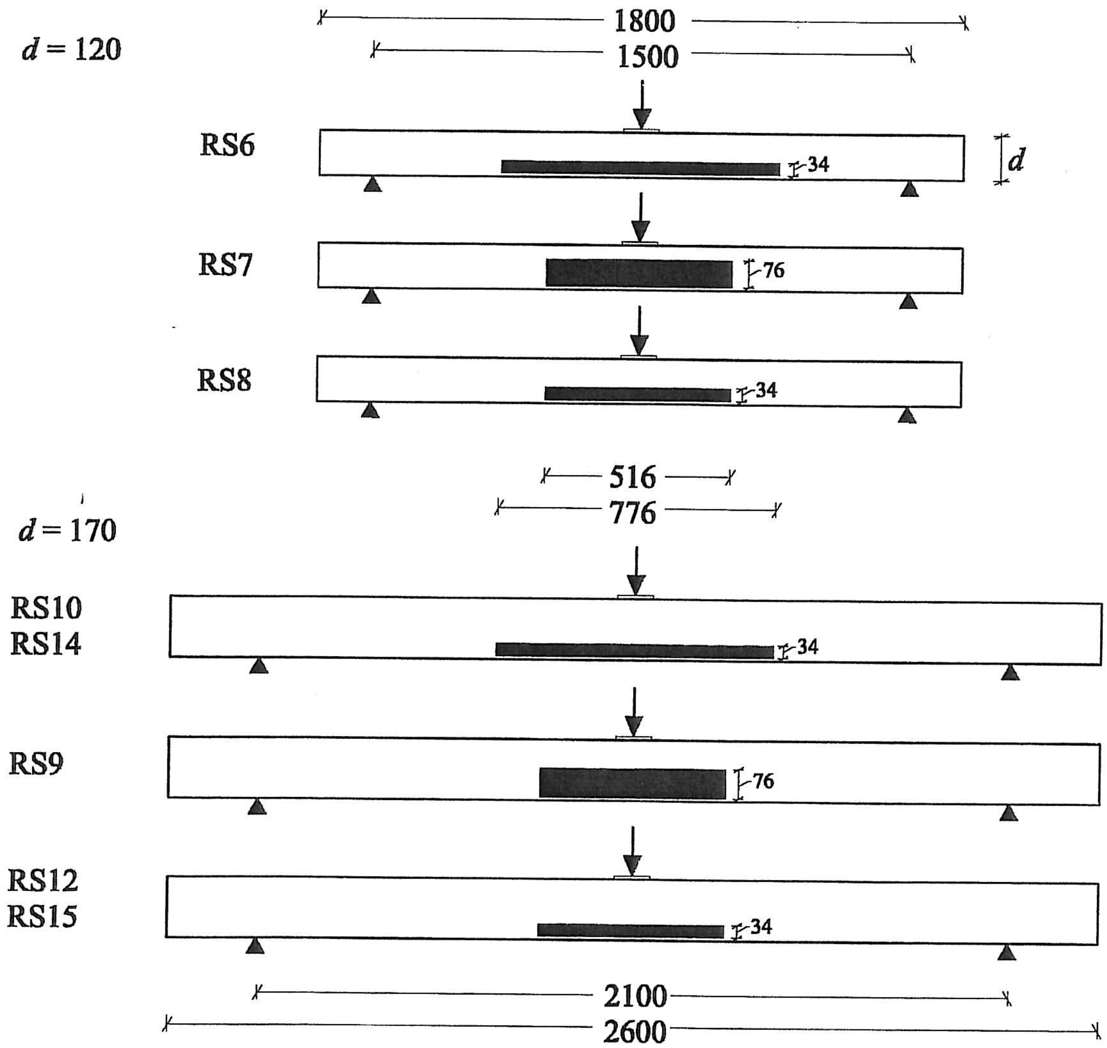


Figure 4: Test specimens with plate reinforcement. Dimensions in mm.

3 Results and discussion

In table 2 the failure loads, failure types and location of failure in each series are given. The mean value m, the standard deviation s, and the coefficient of variation δ are calculated according to DS/EN 1058.

The average wood density is found between 448 - 507 kg/m^3.

3.1 Failure

In general the failure for all beams with or without reinforcement is very sensitive to the appearance and the size of the knots in the tensile side in the area of the load supply.

Series no.	No. of tests	Timber grade	Reinforc. plate		Failure							
					load			type			location	
			w	l	m	s	δ	T	K	S	I	O
d =120mm			mm	mm	kN	kN	%					
RS1	10	K18	-	-	12.41	2.94	24	1	9		8	2
RS6	10	K18	34	776	16.42	3.04	19	3	7		9	1
RS7	10	K18	76	516	15.32	2.46	16	5	5		8	2
RS8	10	K18	34	516	15.02	2.80	19	4	6		5	5
d =170mm												
RS3*	20	K18	-	-	21.15	3.68	17	9	8		12	5
RS5	10	K24	-	-	20.17	4.32	21	6	3	1	9	1
RS9	10	K18	76	516	19.94	3.67	18	5	5		6	4
RS10	10	K18	34	776	20.23	2.97	15	3	5	2	9	1
RS12	10	K18	34	516	21.80	5.62	26	2	5	3	7	3
RS14	10	K24	34	776	22.78	1.69	7	6	2	2	8	2
RS15	10	K24	34	516	22.66	3.44	12	4	3	3	8	2

T: Tensile failure of the fibres. No knot observed at failure.
K: Tensile failure started at a knot.
S: Shear failure in the timber. A crack starts at or below the reinforcement and runs to the beam end parallel to the fibre.
I: Failure started *within* the reinforcing area. In series with no reinforcing, failure started close ($\pm$0.2 m) to the force.
O: Failure started *outside* the reinforcing area or >0.2 m from the load.
* Some failures not observed.

Table 2: Failure load, number of failure types and location of failure in each series.

In table 2 it is seen that the failure loads for the reinforced series with d=120 mm are increased between 20 to 30% compared to the series without plate (RS1). The reinforcing effect is most distinct with long reinforcement plates. In the series with 776 mm reinforcement (RS6) most of the failure occurred within the area of the reinforcement. In series RS7 and RS8 almost half of the failures occurred outside the reinforcement area. The tests with failure outside the reinforcement are caused by the "flat" variation of the moment, see figure 5. In general the moment outside the reinforcing area should be low compared to the moment within the reinforcing area.

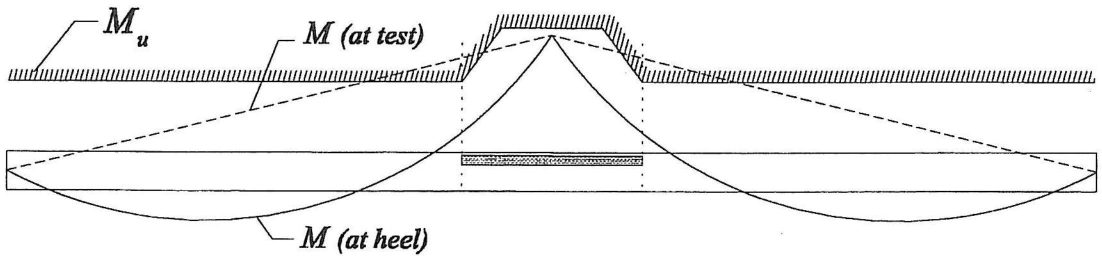


Figure 5: Variation of sectional moment with peaks (M) in relation to the moment capacity (M_u) of a reinforced beam.

Tests with a failure location outside the reinforced area may distort the effect of the reinforcement and they should not be included in the calculation of the average failure load. However, the average failure load of the tests with failure located within the reinforced zone only varies between $\pm$ 1% of the values given in table 2. In series RS7 and RS9 the average failure values are increased by ~5%.

In a few cases total tensile failure in the plates occurred, see figure 7, and in other cases local tensile failure was observed. The decreased coefficient of variation in tests with a reinforcement shows that the failure load is less sensitive to weak sections.

The increasing effect of the reinforcement on the failure load in the series with d=170 mm did not take place. In series with long plates (RS10), where the effect is assumed to be most distinct, the failure loads are even less than the failure loads of the beams without a reinforcement. The coefficient of variation in the series is almost the same. Failure within the reinforcing area causes full tensile failure in the steel or in very few cases withdrawal of the teeth.

An examination of the failure types in the tests with a low failure load compared to the mean value of each series shows that the failure in many cases occurred at knots within the reinforcing area. As it was expected that the reinforcement would have an effect in exactly these cases this indicates that the tensile strength of the plate is too low for a cross-sectional depth of 170 mm. A plate with an increased tensile capacity compared to the GNA 20 S may have a larger effect on the moment capacity of the beam. However, if the reinforced beams shall be an option for practical truss manufacturing it is required that the reinforcement is made by rather small strips (w < 40-50 mm) of plate as the area of a large reinforcing plate may conflict with a beam connector plate, see figure 6.

Visually some of the beams in series planned for K18 may be graded in K24. This assumption is supported by the results from series RS3 and RS5. The mean value of the series with K18 (RS3) are found to be lower than the same value in series with K24 (RS5).

In some tests in RS10, RS12, RS13 and RS15 the failure was caused by shear failure in the wood. This failure type is most distinct in tests with a reinforcement. A crack stated just below the plate and extended to the end of the beam. The size of the

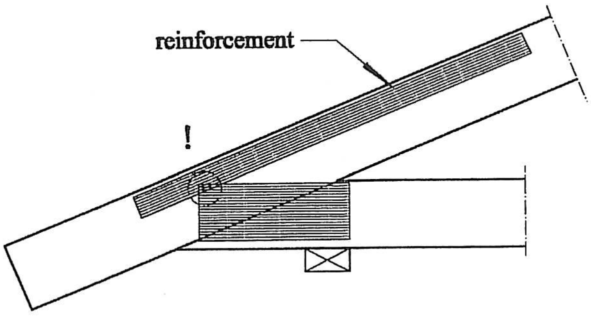


Figure 6: A reinforcement plate in conflict with a beam connecting plate.

failure load for this failure type is found to be rather high (between 20.5kN and 30kN, m=25,3kN).

3.1.1 Embedding at the loading area

In many specimens the 100 mm loading head at the centre of the beam was more or less embedded in the beam. In a few cases the indentation into the wood was more than 10 mm. The embedding is caused by the force perpendicular to the grain. Some failure lines were observed at the load head, see figure 7. The crushing of the wood is most distinct in areas with no knots at the loading head. Crushing perpendicular to

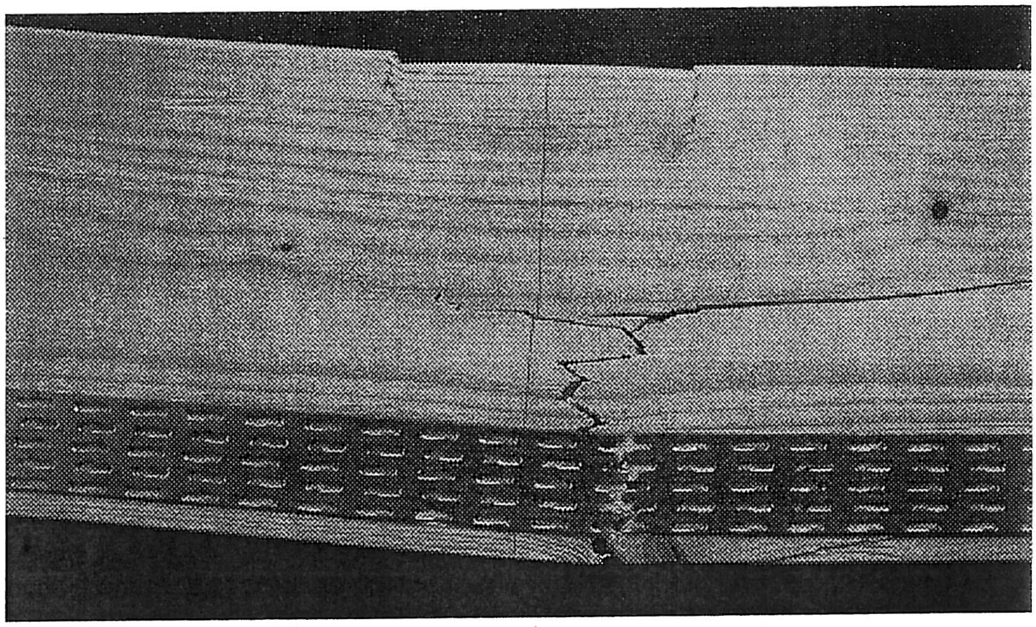


Figure 7: Lines with compression at the loading head and full tensile failure in plate.

the grain can be avoided by increasing the width of the loading head, however, this will cause the moment variation and the moment peak to be less distinct.

The appearance of the moment peak in top chord sections at heel joints will be affected by the size of the contact zone between the top chord and the wedge/ bottom chord, see figure 2. The extent of the contact zone parallel to the chord is hardly bigger than 100 mm. Therefore, it is believed that crushing perpendicular to the grain also appears in chords with moment peaks close to the failure moment. Some analyses on the contact zone are given in Nielsen 1996.

The crushing at the loading head can also be avoided by increasing the span, however this will also cause the moment peak to be less distinct.

3.2 Load-displacement curves

In figure 8 the load-displacements curves for the tests in series RS1 and RS6 are shown. The "O" and "X" denote the maximum load in series RS1 and RS6, respectively.

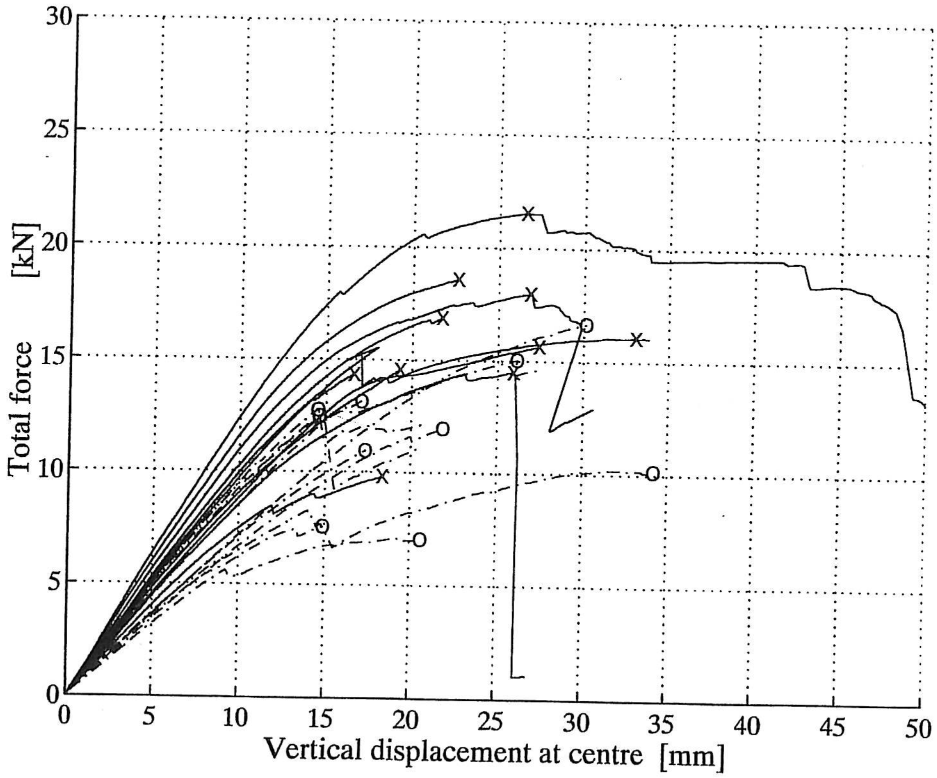


Figure 8: Load-displacement curves of RS1(dash,O) and RS6(solid,X).

In figure 8 it is seen that the initial stiffness of the reinforced beams is increased compared to the non-reinforced beams. The reinforcement changes the properties of the failure from a brittle failure to be slightly more ductile. The displacements at maximum load are almost unchanged.

In a few tests a negative increment of the displacement is observed. This is caused by fact that the displacement measuring pins "jump" out of the measuring points when a sudden crack arise.

In general the effect on the stiffness is very much the same as found for the strength. The stiffness is increased by 20% to 30% for reinforced series with d=120mm and the effect is vanishing for the reinforced series with d=170mm.

4 Comparison with numerical model

Since 1992 a numerical beam model for trusses with punched metal fasteners has been developed and implemented in the MATLAB environment. The programme is called TRUSSLAB. The theory behind the model is given in Foschi(1977), Nielsen(1996) and Ellegaard et al.(1999). TRUSSLAB is in still progress by calibrating the model to fit experimental test results for several different types of tests. So far the model has been compared with tests on splices in tension and bending, Nielsen (1996). The model is able to predict the load-displacement curve even with contact between the timber members and/or plastic conditions in the nail plate. The intention is to develop a model which can estimate the stiffness and the failure in arbitrary joints with punched metal plate fasteners.

4.1 The model

In figure 9 a model of the tests in series RS6 is shown.

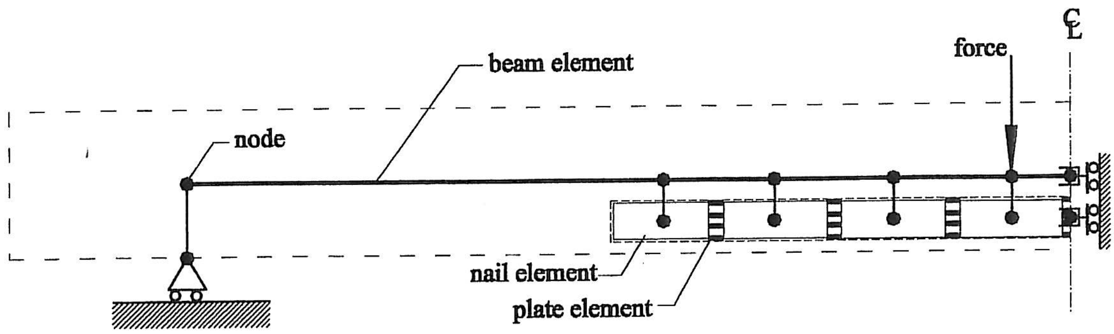
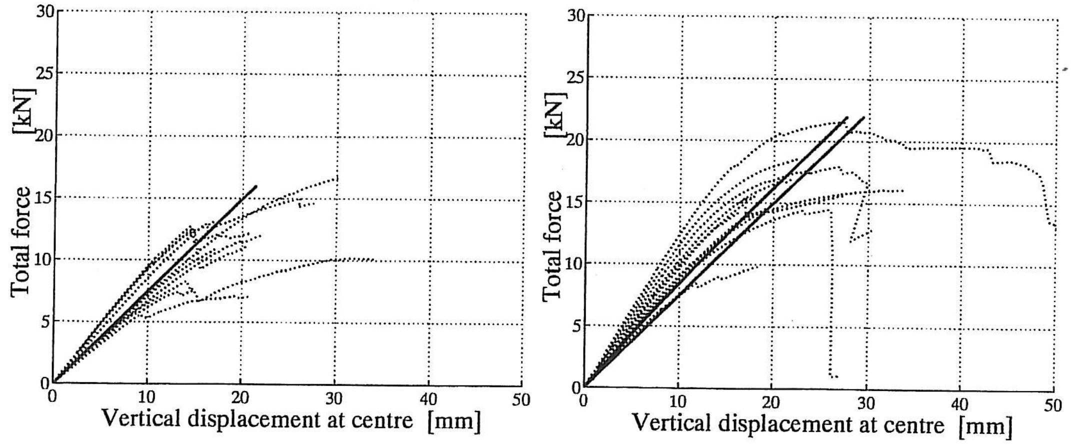


Figure 9: Model of the tests in series RS6.

As the problem is symmetric, only the left half is modelled. For the timber beam Timoshenko beam elements are used and the reinforcement is divided into 4 nail elements connected by 4 plate elements. By dividing the plate into several elements the extent of the plastic zones can be followed. Small auxiliary elements are used to transfer the forces between the beam elements and nail elements. Further description of the the model and the properties of plate and nail elements is given in Nielsen(1996) and Ellegaard et al.(1999).



(a) RS1 (b) RS6

Figure 10: Numerical and experimental load-displacement curves.

4.2 Results and discussion

In figure 10 the numerical load-displacements curves are compared with the experimental results.

In figure 10(a) it is seen that the load-displacement curve of the numerical model with average wood properties according to DS413 (E=9000MPa, G=600MPa) estimates the load-displacement curves of tests. It is assumed that the same stiffness properties can be used to estimate the load-displacement curves of series RS6.

In figure 10(b) it is seen that the load-displacement curve of the numerical model overestimates the displacements. The stiffness of the model is not very sensitive to a variation of the plate location or the plate stiffness. An explanation of the overestimation may be that the wood stiffness of the tests specimens in series RS6 is larger than expected.

According to the model there is a plastic state in the all plate elements. In the two plate elements close to the centre the whole plate is plastic. The plate becomes plastic from 6 kN.

5 Conclusion

Based on the tests the following conclusions can be made:

- The failure load and the stiffness of 120 mm beams with reinforcement are increased by 20 to 30%.
- The reinforcement has no effect on 170 mm beams.

In further analyses it is recommended to:

- use a plate with high tensile capacity (thickness > 1 mm) in beams with a cross-sectional depth of 170mm.
- locate the reinforcement closer to the tensile edge of the timber (< 10 mm).
- limit the width of the reinforcement (< 40 - 50 mm)
- test beams with other cross-section depths and load conditions.

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